Eng. &Tech. Journal, Vol.31, Part (A), No.13, 2013

Ordinary and Encased Stone Columns with Two Different Relative Densities

Dr. Namir K. S. Al-Saoudi
Building and Construction Engineering Department, University of Technology/ Baghdad
Namirks@yahoo.com
Dr. Mohammad M. M. Al-Kaissi
Building and Construction Engineering Department, University of Technology/ Baghdad
mmalkaissi@yahoo.com
Sarah E. K. Al-Baiaty
Building and Construction Engineering Department University of Technology / Baghdad
Sarahimadz@yahoo.com

Received on: 7/8/2012 & Accepted on: 6/12/2012

ABSTRACT

Stone columns technique is most commonly used in increasing bearing capacity, reduces and controls the compressibility and accelerates the rate of consolidation of soft saturated clay. During the last four decades, the technique has been utilized worldwide and proved successful results. Several modifications have been proposed to increase the efficiency of this technique such as addition of additives, use of special patterns of reinforcements, encasing the stone columns with geonet or geogrid to provide extra confinement that enhances the bearing capacity and reduces the settlement drastically without compromising its effect as a drain.

The present paper focuses on the behavior of soft saturated clay reinforced with ordinary and geogrid encased stone columns. The investigation was performed both experimentally through small scale models and through numerical techniques. The influence of relative density of the back fill material and the presence of the encasement are the main parameters investigated.

Ordinary stone columns revealed an increase of 20% in the carrying capacity when the relative density of the backfill stone aggregates increased from 23% to 71%, furthermore the efficiency of the encasement was more pronounced at lower relative density.

Keywords: Numerical Analysis, Geogrid Encasement, Bearing Improvement Ratio, Soft Clay, Stone Columns, Relative Density

الاعمدة الركامية العادية والمغلفه مع كثافتين نسبيتين مختلفتين

الخلاصة

تعتبر الاعمدة الركامية من اكثر طرق التحسين شائعة الاستخدام في زيادة قابلية التحمل وتقليل الانضغاطيه مع تسريع عمليه الانضمام للترب الطينيه المشبعة الضعيفه.

https://doi.org/10.30684/etj.31.13A.2
2412-0758/University of Technology-Iraq, Baghdad, Iraq

خلال العقود الاربعه الاخيره, تم استخدم طريقه التحسين هذه حول العالم واثبتت نتائج ناجحه. تم ادخال عدة تعديلات أقترحت لزيادة كفاءة الاعمدة الركامية كأضافة انماط تسليح خاصة, او تغليف الاعمدة الركاميه لتأمين حصر اضافي لها والتي بدورها سوف تحسن من قابلية تحمل الاعمدة الركامية وتقلل ايضا من الهطول المحتمل بشكل ملحوظ دون التأثير على وضيفتها الاخرى كمصرف للماء.

يهدف البحث الحالي الى دراسة سلوك الترب الطينيه المشبعة الضعيفه المسلحة بالأعمدة الركامية العادية والاعمدة الركامية المغلفه بالجيوكرد(المشبك البلاستيكي).

حيث اجريت الدراسة بشكل عملي بأستخدام موديل مختبري وبشكل نظري بأستخدام برنامج تحليلي. ان تأثير الكثافة النسبيه لجزيئات الاعمدة الركامية و وجود الجيوكرد استخدمت كمتغرات رئيسية في هذا البحث. وقد اظهرت نتائج الترب المسلحة بالأعمدة الركامية العادية زيادة بمقدار ٢٠% عندما تزداد الكثافه النسبيه لجزيئات الاعمدة الركاميه من ٢٣% الى ٧١%, اضافة الى ان التغليف للأعمدة الركاميه بالجيوكرد كان اكثر كفاءة عند الكثافه النسبيه القليله.

INTRODUCTION

The construction of stone columns involves partial replacement or laterally compaction of unsuitable or loss subsurface soils with a compacted vertical column of stone aggregate. So the improvement of soft soil with stone columns is due to three factors, the first one is the inclusion of a stiffer column materials (such as a crushed stone, gravel, and so alike.) in soft soil, the second factor is the densification of surrounding soft soil during installation of stone column. The third factor is the action as a vertical drain (Guetif et al., 2007). Since the pioneering work by Greenwood (1970), there have been much research based on stone columns, reported in the literature, Hughes et al. (1975); Barksdale and Bachus 1983); Priebe (1995). Balaam and Booker (1981) Lee and Pande (1998).

Several researchers have worked on theoretical, experimental and field studies to understand the behavior of ordinary and encased stone columns. Zahmatkesh and Choobbasti (2010), Shahu and Reddy (2011), investigated the behavior of clayey soil reinforced with stone column group.

The encasement imparts additional confinement of stone column and brings in several advantages, as described by Raithel et al.,(2002), Alexiew et al.,(2005), Murugesan and Rajagopal (2006a,b,2007a,b,2008,2009,2010), Keyhosropur et al., (2011), have evaluated the behavior of ordinary stone columns without encasement and geosynthetic encased stone column through experimental and numerical analysis. Ayadat and Hanna (2005) have reported the benefit of encasing the stone columns installed in collapsible soil, Malarvizhi and Ilamparuthi (2004, 2005), brought out the effect of stiffness of encased material on the performance of stone column. Malarvizhi and Ilamparuthi (2007), studied the behavior of the encased stone columns stabilized bed experimentally and numerically.

EXPERIMENTAL WORK
Materials Used
The Soil

A brown clayey soil was brought from a site east of Baghdad. Standard tests were performed to determine the physical and chemical properties of the soil, details are given in Table (1).

Grain size distribution of soil used revealed 3.3 % sand, 31.7% silt and 65% clay as shown in Figure(1). According to USCS, the soil is classified as CL.

The Crushed Stone

The crushed stone material was obtained from a crushing stone factory. It is produced as a result of crushing big stones brought from Penjwen city located in northern part of Iraq. The crushed stone is of white color, angular in shape. Particle size distribution is shown in Figure (2). The crushed stone is of a uniform size, and has considered as poorly graded gradation. The physical properties are presented in Table (2).

Table (1) Physical and chemical properties of natural soil used.

No.	Index property	Index value
1	Natural water content %(wc)	3.1
2	Liquid limit %(LL)	42
3	Plastic limit %(PL)	19.5
4	Shrinkage limit %(SL)	14.2
5	Plasticity index %(PI)	22.5
6	Optimum moisture content (O.M.C)	16
7	Max dry density(M.D.D)	17.15
8	Activity (A_t)	0.60
9	Specific gravity (Gs)	2.69
10	Gravel (larger than 2mm)%	0
11	Sand (0.06 to 2mm)%	3.3
12	silt (0.005 to 0.06mm)%	31.7
13	Clay (less than 0.005mm)%	65
14	Gypsum content %	2.92
15	Total dissolved salt TDS %	3.7
16	SO_3 content %	1.8
17	Organic matter O.M %	0.73
18	Ph value	9.32
19	Classification (USCS)	CL

Note: all tests were performed according to the ASTM (2003).

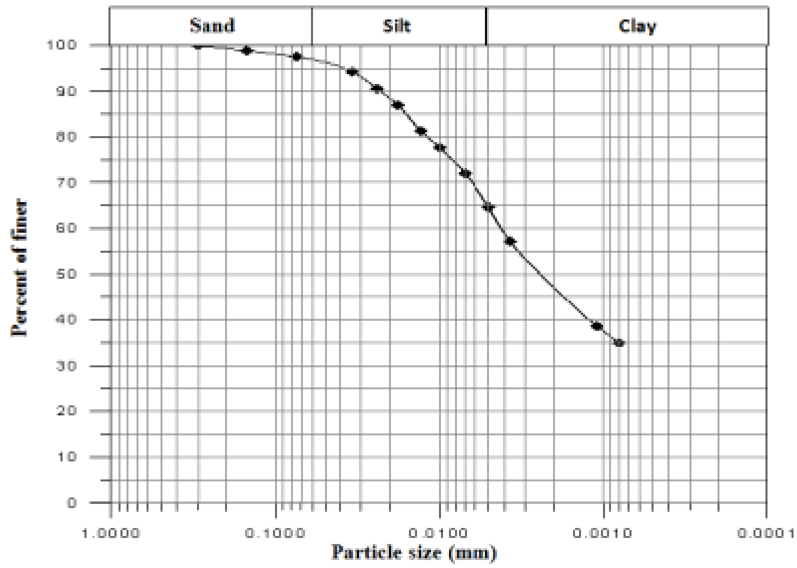


Figure (1) Grain size distribution of soil used.

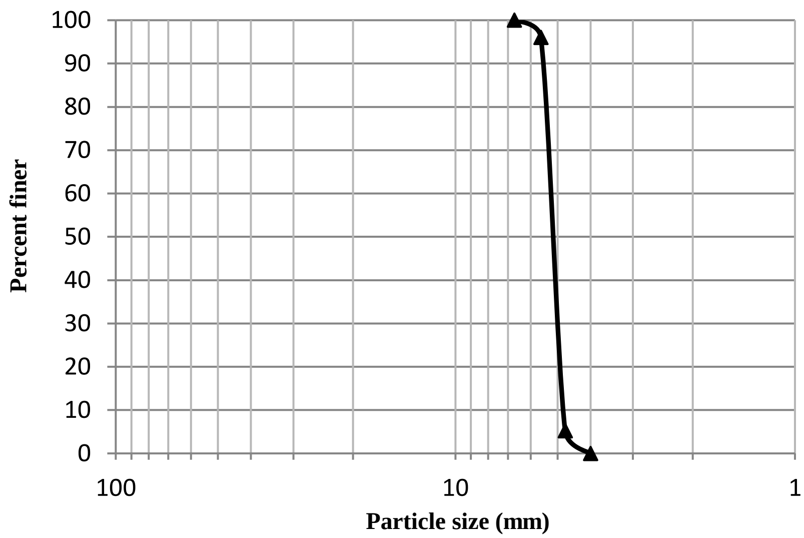


Figure (2) Grain size distribution of crushed stone used.

Table (2) Physical properties of the crushed stone used.

No.	Index property	Index value
1	Max. dry unit weight (kN/m^3)	18
2	Min. dry unit weight (kN/m^3)	12
3	Dry unit weight (kN/m^3) at R.D=71% and 23% respectively	15.7 and 13
4	D_{10} (mm)	4.9
5	D_{30} (mm)	5.0
6	D_{60} (mm)	5.2
7	Specific gravity (Gs)	2.62
8	Coeff. Of uniformity (C_u)	1.06
9	Coeff. of curvature (C_c)	0.98
10	Angle of internal friction (ϕ^0) at R.D = 71% and 23%	42 and 35*

***The angle of internal friction has been estimated using (Das, principles of foundation engineering, 2007).**

GEOGRID REINFORCEMENT

The geogrid, used in the tests, is manufactured by Al-Latifia factory for plastic mash,having engineering properties shown in Table 3 as provided by the manufacturing company.

Table (3) Engineering properties of geogrid used.

Property	Test Method	Unit per (m) length	Data*
Tensile strength at 2 %	ISO 10319	kN/m	4.3
Tensile strength at 5 %		kN/m	7.7
Peak tensile strength		kN/m	13.5
Yield point Elongation		%	20.0
Aperture size		mm*mm	6*6
Thickness		mm	2
Mass per unit area		g / m^2	363

***Determined in accordance with Saudi Arabian Standard Organization (SASO) Procedures.**

PREPARATION OF MODEL TEST

♦ Preparation of Bed of Soil.

The test was conducted at liquidity index of 0.3 corresponding to c_u=15kPa. Natural soil was mixed with enough quantity of water to get the desired consistency. The mixing operation was conducted using a large mixer manufactured for this purpose. After thorough mixing, the wet soil was kept inside tightened polythen bags for a period of two days to get uniform moisture content. After that, the soil was placed and compacted in a steel container (1000×400×700)mm in ten layer, each layers was leveled gently using a wooden tamper, then the leveled layer was tamped gently with a manufactured metal hammer of 9.87 kg and dimension of (150×150)mm in order to remove any entrapped air.

This process continues for the ten layers till reaching a thickness of 500 mm of soil in the steel container. After completing the final layer, the top surface was scraped and leveled to get as near as possible a flat surface, then covered with polythen sheet to prevent any loss of moisture.

A wooden board of similar area to that of the surface area of bed soil (400×700) mm was placed on the bed, then a setting pressure of 5 kPa was applied. The bed was left for a period of two days to regain part of its strength.

♦ Formation of Ordinary Stone Columns

The construction procedure of the stone columns starts directly after the preparation of the bed of soil. The depth of each stone column was predetermined (corresponding to L/D=6). A PVC pipe with external diameter of 50 mm was pushed down the bed to the specific depth with the aid of the frame. To remove the soil inside the PVC pipe, a hand auger, manufactured for this purpose was used. After that the PVC pipe was removed carefully. The stones were carefully charged into the hole in five layers and compacted using 44mm diameter rod to achieve a density of 15.6 kN/m^3 by a tamping rod, and a density of 13kN/m^3 by just adding the stone particles with slightly press into the hole using the same rod. All the stone columns have a diameter of 50mm, length to diameter L/D =6, spacing between stone columns and area replacement ratio a_r (The area replacement ratio is the ratio of the granular pile area over the whole area of the equivalent cylindrical unit cell) are shown in Figure (3).

♦ Formation of Encased Stone columns

To install the encased stone columns, the same procedure of the construction of the ordinary stone columns was followed here. First formed samples geogrid tubes were made by warping up roll of geogrid and sew by a nylon strings with the diameter 48mm and length of the encased stone column L/D = 6. Then construction procedure of the encased stone column started after the preparation of the bed of soil.

The geogrid tube was inserted into the stone column hole using the PVC pipe. The crushed stone was poured into the hole in layers and compacted gently by tamping rod.

Figure (3) Stone Columns Configuration Details.

EXPERIMENTAL SETUP

The original setup was manufactured by Rahil (2007), to study the behaviour of soft clay reinforced by stone columns underneath a railway track. Track with the length of 2000mm interacting to form a continuous footing of that width was modeled by a plain strain 200mm wide footing as shown in Figure (4), capable of applying both of monotonic and repeated loading. An extensive development process for the apparatus was carried out regarding the accuracy of testing, pressure range and programmable logic control.

Those developments helped in recording the outcomes of the tests in a more accurate form.

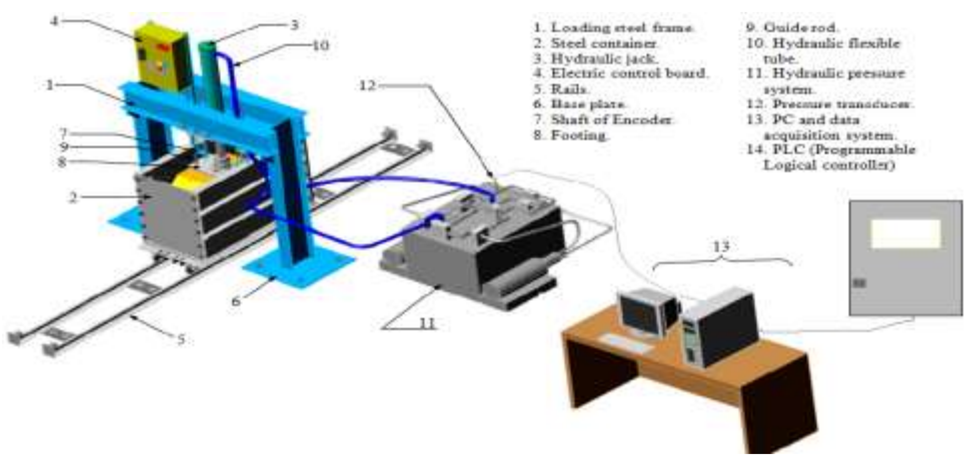


Figure (4) the general view of the apparatus.

Model Test

After the completion of the preparation of the bed of model housed inside the steel container, with and without treatment, the container was then moved along the rails and fixed in position in such a manner that the center of the footing coincided with the center of the bed of the model. The footing was then brought in contact with top surface of the bed of the model. The monotonic loading was applied gradually through the hydraulic jack which operates at a controlled displacement of 0.05mm/sec. The process continues up to failure.

RESULT AND DISCUSSION

Model Test Results of Untreated Soil

This test was conducted on bed of untreated soil with undrianed shear strength ranging between 15-20kPa. This test is considered as a reference to obtain the degree of improvement gained after introducing any type of improvement. Figure (5), represents the relationship between the applied vertical stress q kPa versus settlement S mm.

The corresponding results obtained from F.E.M (Plaxis 3D Foundation) are also presented. The soil is modeled with 15 nodes triangular finite element, and coarse mesh was refined as shown in Figure (6), where the non-linear behaviour of clay is treated as undrianed material and modeled using hardening soil model as illustrated in Table (4).

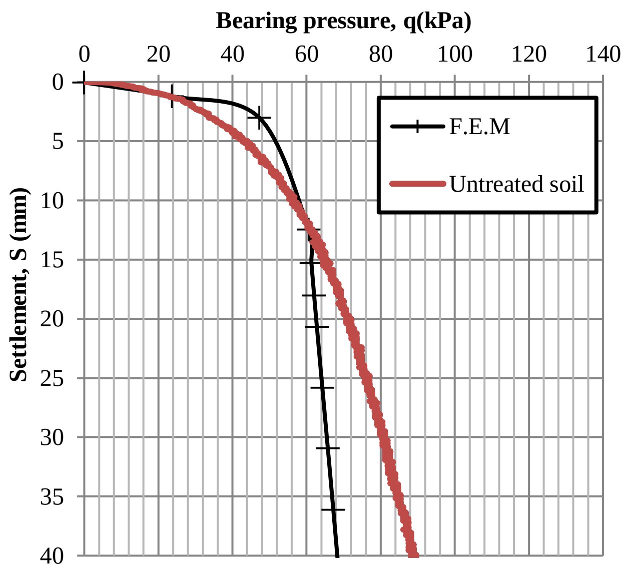


Figure (5) Bearing pressure versus settlement for untreated soil.

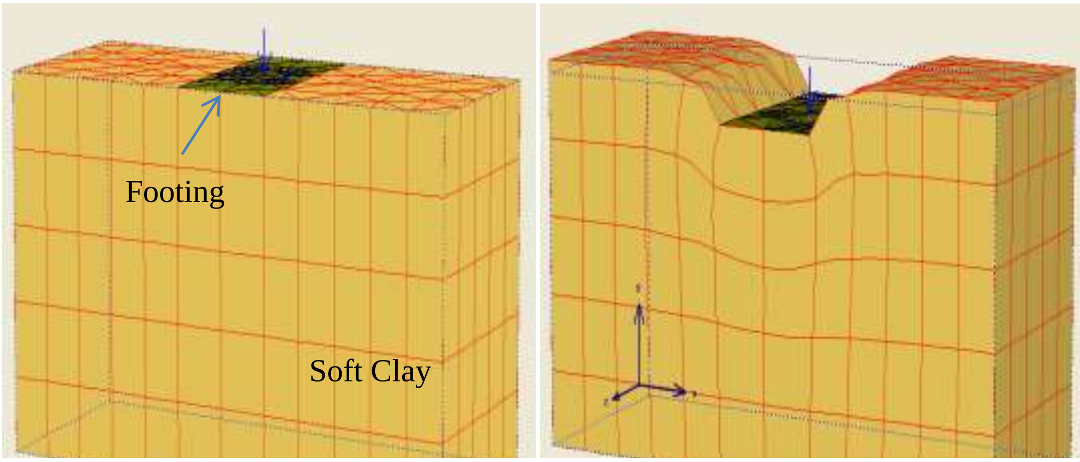


Figure (6) Finite element mesh for tested model.

Table (4) Parameters used for soil modeling.

Parameter	***Clay***
Model	Hardening soil model
E(kPa)	3000
ν	0.45
γ_{sat}(kN/m^3)	19
γ_{unsat}	15.5
Ø	Not required
c_u(kPa)	15
ψ	0

A remarkable divergence is observed between the model test and F.E.M curves not exceeding 20% started from 62 kPa at 15mm. The mode of failure is close to the general shear pattern. The bearing capacity at failure corresponding to S/B=10% (20mm) are 72.1kPa and 62.7kPa for model test and F.E.M analysis respectively. Figure (7) shows the bearing ratio q/c_u plotted against settlement ratio S/B%.

The bearing ratio q/c_u at failure is 4.0 corresponding to the settlement ratio S/B of 10%. This value is within the acceptable range of bearing capacity factor N_c ranging from 4 to 6.28 for saturated soil at undrained condition with Ø=0.

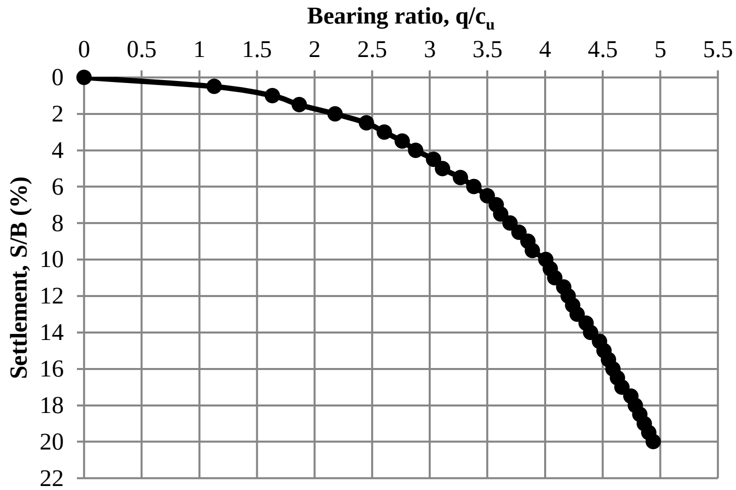


Figure (7) Bearing ratio versus settlement ratio for untreated soil.

Model Test Results of Soil Treated With Ordinary Stone Columns

Figures (8) a,b show the relationship between the vertical stress q kPa plotted against the settlement S mm for the two relative densities. The corresponding results obtained from F.E.M analysis are also presented by using hardening soil model and Mohr-Coulomb criterion for soil and stone column respectively, as shown in Table (5). The non-linear behaviour of soil is treated as undrianed material and the elasto-plastic bahaviour of stone columns is modeled as a drained material. The remarkable convergence observed between the corresponding model test and F.E.M analysis continues to approximately 20mm settlement then followed by slight divergence between the two curves.

Table (5) Parameters used for material modeling.

Parameter	Ordinary stone columns at 23%	Ordinary stone columns at 71%
Model	Mohr-Coulomb	Mohr-Coulomb
E(kPa)	7000	17000
ν	0.3	0.3
γ_{sat}(KN/m^3)	14.5	16.5
γ_{unsat}	13	15.7
Ø	35	42
c_u(kPa)	0.01	0.01
ψ	15	20

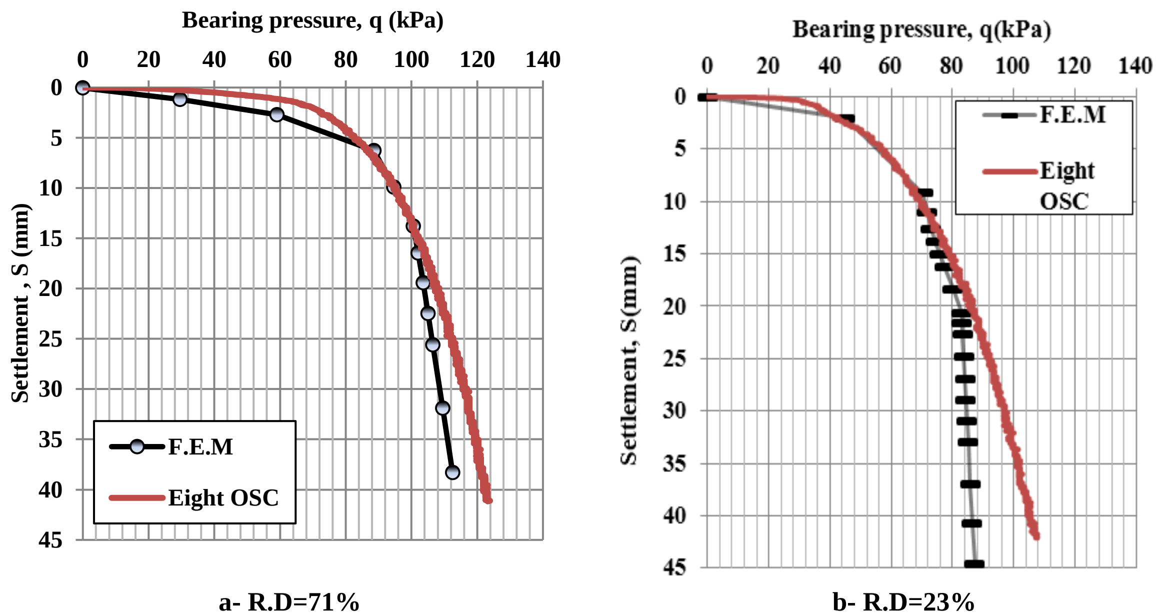


Figure (8) Bearing pressure versus settlement for model test with OSC.

Figure (9) represents the relationship between the bearing ratio plotted against the settlement ratio. Results of untreated soil are also presented for comparison purpose. The results of this Figure illustrates that a substantial reduction of approximately 20% in bearing ratio q/c_u is observed as the relative density of stone columns decreases from 71% to 23%. The bearing ratios at failure are 5.9 and 4.8 for soil treated with ordinary stone at 71% and 23% relative density respectively. So, the insertion of stone columns into weak soils is not just a replacement operation, it is believed that the stone column can change in both the material properties and state of stresses in the treated soil mass, as reported by (Guetif et al., 2007). Relative density of stone columns plays a major role to increase the strength of composite system and thus the bearing capacity of ground increase, as reported by (Shahu and Reddy, 2011).

The variation of bearing improvement ratio q_t/q_{unt} versus settlement ratio S/B% is shown in Figure 10. Peak values are observed at nearly S/B=0.5% followed by a rapid drop in the bearing improvement ratio. This behaviour is due to the fact that the stone columns are stiffer than the surrounding soil. As model is loaded, the stress is transferred to the stone columns expressing these peak values then it is gradually transferred to the surrounding soil implied by the drop in the improvement ratio.

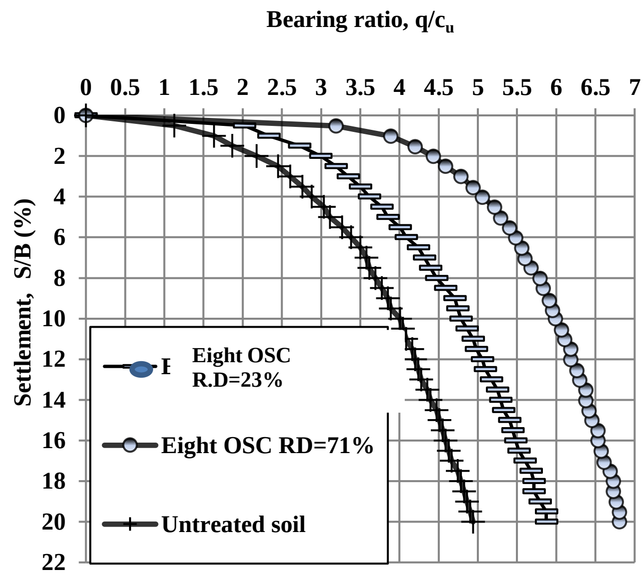


Figure (9) Bearing ratio versus settlement ratio for model tests with ordinary stone column.

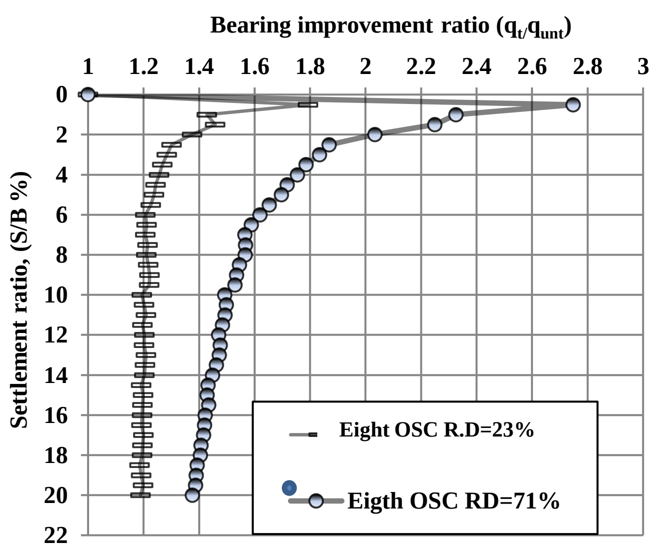


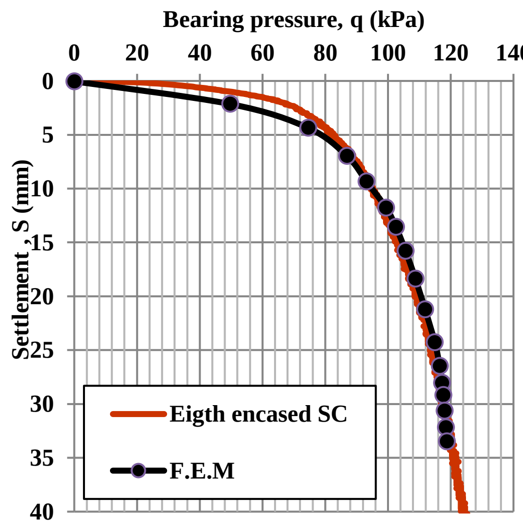
Figure (10) Bearing improvement ratio versus settlement Ratio for model tests with ordinary stone columns.

Model Test Results of Soil Treated With Encased Stone Columns

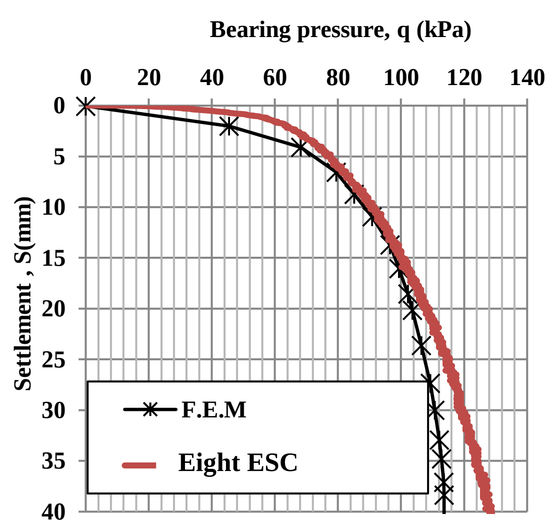
Figure (11 a,b) represent the relationship between vertical stress q kPa plotted against the settlement (mm) for each treated configuration . F.E.M analysis was also carried out to check and understand the behaviour of encased stone columns. The experimental results showed good conformity with F.E.M analysis. The bearing pressure-settlement behaviour of F.E.M is obtained using hardening soil model, Mohr-Coulomb and linear elastic model for soft soil, stone column and geogrid encasement respectively as shown in Table (6). The behaviour of geogrid is treated as a wall element modeled using the linear elastic model. To provide the confinement around the encased stone columns, circular stone columns area was equivalent to square area.

Table (6) Parameters used for material modeling.

Parameter	Encased stone columns at 23%	Encased stone columns at 71%
Model	Mohr-Coulomb	Mohr-Coulomb
E(kPa)	17000	20000
ν	0.3	0.3
$\gamma_{sat}(KN/m^3)$	14.5	16.5
γ_{unsat}	13	15.7
Ø	35	42
c_u(kPa)	0.01	0.01
ψ	15	20



a- R.D=71%



b- R.D=23%

Figure (11) Bearing capacity versus settlement for model test with encased stone columns.

Figure (12) shows the variation of bearing ratio q/c_u versus settlement ratio S/B% for soil treated with encased stone columns. Results of untreated soil and soil treated with

OSC for both relative densities 71% and 23% are presented for comparison purpose. Based on the results, two main factors play a major role of increasing the bearing capacity of soil treated with encased stone columns, the first one is the relative density of the backfill stone particles, and the second factor is the resistance of the surrounding soil (undrianed shear strength of the soft soil).

It demonstrates no significant increase in bearing capacity for soil treated with ESC prepared at 71% relative density as compared with OSC. This may be due to the high resistance developed through the dense packing of the stone particles at relative density 71%. The hoop tension mobilization will not take place and equilibrium state may occur between the lateral support provided by the surrounding soil and the lateral support provided by the geogrid encasement (Raithel et al., 2005).

A remarkable convergence is observed between models tested with ESC prepared at relative density 23% with both models tested with ESC and OSC prepared at relative density 71%. This may be due to the influence of the relative density of backfill stone particles on the encasement efficiency. The compression of stone columns under loading effect was mainly due to the readjustment of stone particles, or slippage over each other. The geogrid encasement helps of easy formation of the stone columns particles and improving its strength and stiffness.

The hoop tension mobilized of the encasement revealed approximately 25% increase in bearing ratio as compared with model tested with OSC prepared at the same relative density of 23%.

Summarized values of bearing ratio at failure for models tested with ESC at 71% and 23% relative density of the same configuration are shown in Table (7).

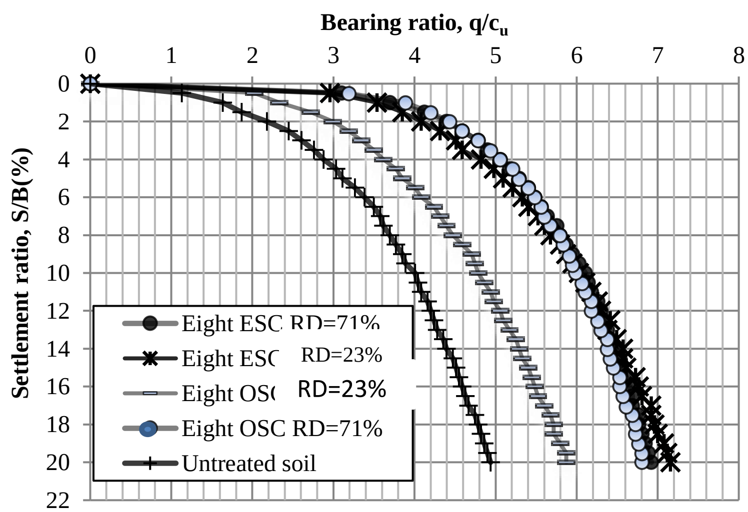


Figure (12) Bearing ratio versus settlement ratio for model test with encased stone columns.

Table (7) Bearing ratio at failure for model tested with encased

stone columns at 71% and 23% R.D.

Cases	Bearing ratio at failure q/c_u	
	OSC	ESC
stone column at R.D=71%	5.9	6.03
stone column at R.D=23%	4.8	6

To evaluate the amount of improvement ratio achieved by the encased stone columns over untreated soil and soil treated with ordinary stone columns at 23% and 71% relative density, q_t/q_{unt} versus settlement ratio S/B% are presented in Figure (13). The results demonstrate that the bearing improvement ratio increases rapidly to peak values at settlement ratio S/B= 0.5%, then drops down and ultimately reached to constant value of settlement ratio. Model tests with ESC and OSC at relative density 71% exhibited very close behaviour with model tested of eight ESC at 23% relative density.

Summarized values of bearing improvement ratio at failure for models tested with ESC at 71% and 23% relative density are shown in Table (8).

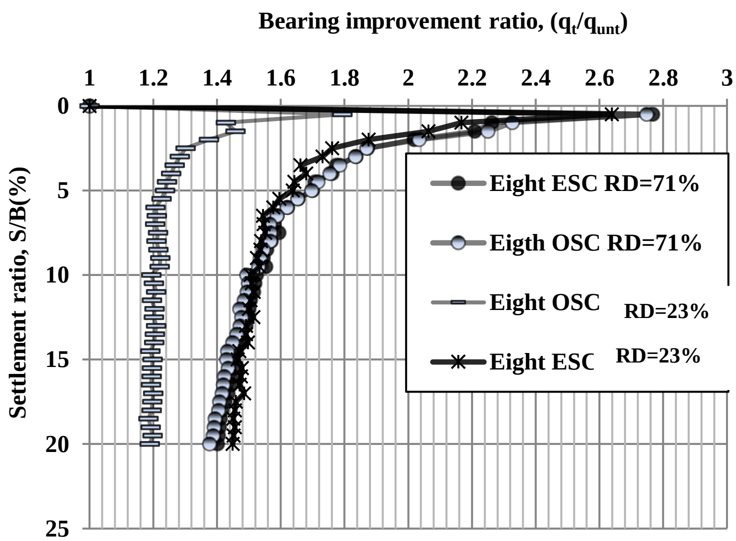


Figure (13) Bearing Improvement ratio versus settlement ratio for model test with encased.

Table (8) Bearing improvement ratio at failure for model tested with encased stone columns at 71% and 23% R.D.

Cases	***Bearing improvement ratio at failure(q_t/ q_{unt})***	
	OSC	***ESC***
stone column at R.D=71%	1.49	1.5
stone column at R.D=23%	1.19	1.5

CONCLUSIONS

1- The mode of failure for soil with c_u ranging between 15-20 kPa is close to general shear failure and the bearing ratio q/c_u obtained is within the acceptable range of bearing capacity factor N_c ranging from 4 to 6.28 for saturated soil.

2- Increasing the relative density of backfill stone particles from loose state 23% to dense state 71% provided approximately 20% increasing in the strength of the composite ground.

3- With the range of undriand shear strength $c_{u\,=}$15-20 kPa, the efficiency of the encasement is highly effective at 23% relative density of the backfill material.

4- Reducing the relative density of backfill material of stone particles from dense 71% to looses 23%, revealed a significant effect of the encasement on the behaviour of stone columns. The increasing in values of bearing ratio q/c_u and bearing improvement ratio q_t/q_{unt} reached to 1.25 times, as compared with OSC.

5- The F.E.M results from Plaxis 3D Foundation are in close agreement with model test results in the pre failure range of stress. As stress increase to failure or post failure the discrepancy between the two approaches become significant.

REFERENCES

[1].Alexiew, D., Brokemper, D. and Lothspeich, S., (2005), "Geotextile Encased Columns (GEC): Load capacity, geotextile selection and predesign graphs.", Geotech., Spec. Pub No. 130–142, 497-510.

[2].Ayadat, T. and Hanna, A. M., (2005), "Encapsulated stone columns as a soil improvement technique for collapsible soil." Ground Improv., 9(4), 137.

[3].Balaam, N.P. and Booker, I.R., (1981),"Analysis of rigid rafts supported by granular piles.", International Journal for Numerical and Analytical Methods in Geomechanics 5, 379-403.

[4].Barksdale, R.D. and Bachus, H.G., (1983), "Design and construction of stone columns.", Dept. of Transpiration, Federal Highway Administration Report , USA report No. FHWA/RD 83/026.

[5].Das, B.M., (2007)," Principles of foundation engineering." 6th Edition, International Student Edition, United State

[6].Greenwood, D.A., (1970)," Mechanical improvement of soils below ground Eng.", Conf, Institute of Civil Engineers, June, pp 11-22.

[7].Guetif, Z., Bouassida, M. and Debats, J. M., (2007), "Improved soft clay characteristics due to stone column installation.", Comput. Geotech., 34(2), 104–111.

[8].Hughes, J.M.O., Withers, N.J. and Greenwood, D.A., (1975),"A Field Trail of the Reinforcing Effect of a stone column in soil.", Journal of Geotechnique, Vol. 25, No. 1, March, pp. 31-44.

[9].Keyhosropur,L., Soroush A. and Imam R., (2011), "A study on the behaviour of a geosynthetic encased stone columns group using 3D numerical analysis.", Geotechnical Conference , Amerkabir University of Technology, Tahran, Iran.

[10].Lee, J.S. and Pande, G.N., (1998),"Analysis of stone-column reinforced foundations.", International Journal for Numerical and Analytical Methods in Geomechanics 22, No.12. 1001-1020.

[11].Malarvizhi, S.N. and Ilamparuthi, K., (2004)," Load versus settlement of clay-bed stabilized with stone and reinforced stone columns.", Proceeding of the 3rd Asian Regional Conference on Geosynthetics, GEOASIA, Seoul, Korea, 322-329.

[12].Malarvizhi, S.N. and IIamparuthi, K., (2005),"Modeling of geogrid encased stone column.", Proceedings of the National Conferences on Geotechics Environmental, Malaysia.

[13].Malarvizhi, S.N. and Ilamparuthi, K., (2007),"Comparative study on the behavior of encased stone column and conventional stone column.", Soils and Foundations Journal 47, No. 5, 873-885.

[14].Murugesan, S. and Rajagopal, K., (2006a),"Geosynthetic-encased stone columns: Numerical evaluation.", Geotext. Geomem., 24(6), 349–358.

[15].Murugesan, S. and Rajagopal, K., (2006b), "Numerical analysis of geosynthetic encased stone column.", Proc., 8th Int. Conf. on Geosynthetics, Yokohama, Japan, 1681–1684.

[16].Murugesan, S. and Rajagopal, K., (2007a), "Model tests on geosynthetic encased stone columns.", Geosynthet. Int., 24(6), 349–358.

[17].Murugesan, S. and Rajagopal, K., (2007b),"Model tests on stone columns subjected to lateral soil movement.", Proc., 13th Asian Regional Conf. on Soil Mechanics and Geotechnical Engineering, Vol. 1, Allied, New Delhi, India, 989–992.

[18].Murugesan, S. and Rajagopal, K., (2008),"Performance of encased stone columns and design guidelines for construction on soft clay soils.", Proceedings of the 4th Asian Regional Conference on Geosynthetics, Shanghai, China, 729-734.

[19].Murugesan, S. and Rajagopal, K., (2009),"Investigations on the behavior of geosynthetic encased stone columns.", Proc. of the 17th ICSMGE, Alexandrina, Egypt.

[20].Murugesan, S. and Rajagopal, K., (2010)," Studies on the behaviour of single and group of geosynthetic encased stone columns.", Journal of Geotechnical and Geo environmental Engineering, 136(1), 129-139.

[21].Priebe, H.J., (1995),"The Design of Vibro-Replacement.", Ground Engineering Journal, Vol.28, No.10, Dec., pp.31-37.

[22].Rahil, F.H., (2007),"Improvement of Soft Clay Underneath a Railway Track Model Using Stone Columns Technique.", Ph.D. Thesis, University of Technology, Iraq.

[23].Raithel, M., Kempfert, H.G. and Kirchner, A., (2002),"Geotextile encased columns (GEC) for foundation of a dike on very soft soils.", Proc., 7th Int. Conf. on Geosynthetics, Swets & Zeitlinger, Nice, France, 1025–1028.

[24].Shahu, J.T. and Reddy, Y.R., (2011)," Clayey soil reinforced with stone column group: Model tests and analysis.", Journal of Geotechnical and Geo environmental Engineering , 137(12), 1265-1274.

[25].Zahmatkesh, A. and Choobbasti, A.J., (2010)," Investigation of bearing capacity and settlement of strip footing on clay reinforced with stone columns.", Australian Journal of Basic and Applied Sciences, 4 (8): 3658-3668.